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Engineering and Design
RELIABILITY ANALYSIS AND RISK ASSESSMENT FOR SEEPAGE AND SLOPE STABILITY FAILURE MODES FOR EMBANKMENT DAMS

1. Purpose

This document provides guidance for performance of risk assessment analyses of dam safety-related detrimental seepage (internal erosion, piping, under seepage, and heave) and slope stability problems. Detailed descriptions of reliability and risk analysis for seepage and slope stability problems are provided.

2. Applicability

This ELT is applicable to all USACE Commands having Civil Works Responsibilities. It applies to all studies for major rehabilitation projects.

3. References

See Appendix A.

4. Distribution

Approved for public release, distribution is unlimited.

5. Discussion

a. Risk assessment is performed to evaluate various parameters to assist in the decision making process. A risk analysis and assessment provides the total annualized consequences or risk with and without the proposed seepage/stability correction project. By comparing the with and without projects, the risk assessment process is used to guide the selection of the alternative that is most effective in reducing the risk of unsatisfactory performance.

b. Site characteristics and potential modes of failure are identified. An event tree is then used to describe the various modes of unsatisfactory performance, and weighted damages are determined by multiplying the probabilities of occurrence and the costs incurred to give expected risk. Once the risk is determined for the without-project condition, the process is repeated for each with-project alternative. The most feasible alternative can then be selected.

c. The above methodology is used to assess seepage risk analysis and slope stability risk analysis.

d. Practical examples and case histories on the application of reliability analysis and risk assessment for seepage and slope stability failure modes of embankment dams are presented in the appendices to this ETL as follows:

-Appendix A lists the references used in this document.
-Appendix B discusses Poisson distribution.
-Appendix C provides a discussion of the six-sigma rule.
-Appendix D is a step-by-step reliability analysis of a slope stability problem.
-Appendix E is guidance on performing expert elicitation.
-Appendices F and G are case histories on applying risk analysis to projects with seepage problems.
-Appendix H provides information on using a historic data model to analysis piping in an embankment dam.
-Appendix I is a case history of a slope stability reliability analysis.
-Appendix J is a case history of historical frequency of occurrence model for pipes.
-Appendix K discusses Monte Carlo simulation.

FOR THE DIRECTOR OF CIVIL WORKS:

11 Appendices
(See Table of Contents)

DONALD L. BASHAM, PE
Chief, Engineering and Construction
Directorate of Civil Works

RELIABILITY ANALYSIS AND RISK ASSESSMENT FOR SEEPAGE AND SLOPE STABILITY FAILURE MODES FOR EMBANKMENT DAMS

Table of Contents

RELIABILITY ANALYSIS AND RISK ASSESSMENT FOR SEEPAGE AND SLOPE STABILITY FAILURE MODES FOR EMBANKMENT DAMS

1. Introduction to Geotechnical Reliability Procedures

a. Requirement For Risk Assessment.

(1) The Office of the Assistant Secretary of the Army for Civil Works directed that the Dam Safety Seepage and Stability Correction Program comply with policy and criteria of the Civil Works Major Rehabilitation Program. A Major Rehabilitation Evaluation Report is required to obtain "Construction, General" (CG) funding for correction of seepage/stability problems at existing dams. Upon approval of the Evaluation Report, a proposed project can be funded in accordance with EC 11-2-179, Section B-2-4.

(2) A risk assessment, as generalized in Figure 1, is required as part of that decision document. The risk analysis and assessment provides the total annual economic risk (for example, economic and environmental impact and loss of life) with and without the proposed seepage/stability correction project. The “without project” or “baseline” condition should demonstrate that without the proposed correction to the project that the expected probability of unsatisfactory performance is high for all loading conditions or increases over time. The "with project" condition should demonstrate a significant reduction in the conditional probability of unsatisfactory performance (and reduced annual economic risk) to show that the project provides an acceptable level of performance for all loading conditions. The risk assessment process should be used to guide the selection of the alternative that is most effective in reducing the annual economic risk of unsatisfactory performance.

(Annual Loading Probability)
X
(Probability of Unsatisfactory Performance)
X
(Conditional Performance Level)
X
(Consequences)
=
Annual Risk for Each Load

Figure 1 - Generic Concept to Determine Annual Risk for a Given Load

(3) Risk is defined as the probability of a loss occurring in a given time period (annually), where loss consists of all economic damages (measured in dollars) and the environmental loss of habitat (habitat units loss) as well as the potential for loss of life.

b. Risk Assessment Process. The required risk assessment process involves two steps, a thorough definition of the problem, and then development of an event tree to provide a framework for analyzing the annual economic risk of various alternatives. These two steps are further described below.

(1) Problem Definition. Risk assessment begins with a definition of the problem. The steps in defining the problem include: 1) Site characterization, and 2) Identification of potential modes of failure. (For the purpose of this discussion, the "site" is defined to include both the natural setting where the dam exists and the dam itself.)

(a) Site Characterization

1 Site characterization consists of identification of all of the site-related factors significant in the evaluation of embankment stability and detrimental seepage. Typical factors include characteristics of the constructed project, such as embankment geometry, zoning, materials, construction methods, and seepage cutoff/control features; and characteristics of the setting, such as site geology and stratigraphy, and foundation material characteristics. Additional site information is provided by documented performance history.

2 To the maximum extent practicable, existing data from design and construction records, performance history, and post-construction investigations are used to characterize the site. At preliminary stages of risk assessment, existing data supplemented by engineering judgment provide a sufficient basis for evaluation. If significant risk for the project is indicated, field investigations and additional analyses are warranted.

(b) Identification of the Potential Modes of Failure. The second step in defining the problem consists of identifying potential modes of failure. For example, with respect to piping, uncontrolled seepage through the embankment, through the foundation, or through the embankment into the foundation can all cause failure of a dam. With respect to embankment stability, sliding or deformation may be the result of insufficient strength of materials, may be induced by excessive pore pressures, or could be induced by an external loading. All potential modes of failure should be identified. Analysis and remediation of seismic or hydrologic deficiencies are covered by ER 1110-2-1155, Dam Safety Assurance Program and are not covered under the Major Rehabilitation program.

(2) Event Trees.

(a) After defining the problem through site study and determination of the modes of failure, risk analysis requires development of an event tree. An event tree is a graphical representation of the various events that could occur (each with estimated probabilities), and in turn, the various events that could follow from each preceding event. Construction of an event tree allows those performing the risk analysis to think through the various sequences of events that can cause unsatisfactory performance of the dam and the consequences resulting from the unsatisfactory performance. A simplified event tree is shown in Figure 2, which depicts four analytical components in risk assessment of a dam. The four components are annual loading frequency, probability of unsatisfactory performance, performance level, and consequences. Risk (in the last column) is the product of the four components.

(b) Traditional design in geotechnical engineering is based on allowable factors of safety developed by the profession as a whole from years of experience. Reliability analysis is based on the capacity-demand model, where the probability of unsatisfactory performance is defined as the probability that the demand on the system or component exceeds the capacity of the system or component. The capacity and demand can be combined into a single function and the event that the capacity equals the demand taken as the limit state. The probability of unsatisfactory performance is the probability that the limit state will be exceeded. The term “probability of unsatisfactory performance” is used instead of “failure” because failure is often thought of as the complete catastrophic failure of the dam with a complete release of pool. This is not necessarily the case. Failure of a dam slope as defined by a limit state analysis resulting in a factor of safety of one could be a massive slope failure causing the complete release of pool. But it could be a lesser slide not causing any pool release but which creates an unreliable situation; it could be a surface slump on the dam, or it could be transverse crack on the crown of the dam. Therefore, the term unsatisfactory performance is used instead of the term failure. The capacity-demand model is represented by Equation 1.

$$Factor\ of\ \ Safety = \frac{Capacity}{Demand} \tag{1}$$

In the capacity-demand model, unsatisfactory performance is based on the probability that the limit state is exceeded meaning the probability that the factor of safety is less than one. Then the different possible unsatisfactory performance outcomes (termed performance levels) are determined along with their probability of occurrence.

(c) The term “performance level” is used to indicate how the structure will physically perform due to an unsatisfactory performance event .

(d) The event tree facilitates the systematic calculation of conditional probabilities of various outcomes and provides the basis for calculating consequences and annual economic risk of proposed alternative actions. Based on this analysis, proposed alternative actions can be rationally compared. Detailed discussion on the terms, development and use of event trees is given in Paragraph 2.

c. Conventional Analysis. Reliability analysis is to be used for risk assessment of an embankment dam. Reliability analysis is not meant to be used in place of conventional analysis for design purposes. The basic premise of reliability models is that they are an investment tool for decision-making. They allow decision makers the ability to analyze the risk and associated consequences of multiple future scenarios by accounting for the uncertainties in the analysis and possible outcomes. Nowhere in geotechnical practice has probabilistic methods supplanted the use of conventional methods of design, such as the factor of safety approach. For routine design, deterministic methods have been proven to provide safe, adequate, and economical designs. Probabilistic design would require an additional work effort that normally would not be warranted. The use of probabilistic design would require a huge calibration effort, which has not been accomplished to date, to make sure that safe economical designs would be produced. This calibration effort would involve performing comparative deterministic and probabilistic analyses for a wide range of geotechnical problems.

2. Event Trees

a. Introduction. An initial step in performing a risk analysis of an embankment dam is to construct an event tree that includes all relevant failure modes. An event tree allows engineers to diagram the steps and conditions that will determine the risk associated with an embankment dam. It also allows them to think through the process of the events that can cause failure of the dam and the consequences resulting from the failure. Once the initial event tree is constructed, a review of the event tree is made to make sure it accurately portrays the events that can cause dam failure and to make sure that no failure modes or events are left out. A simplified example of an event tree is shown in Figure 2 that depicts the steps in the risk analysis of an embankment dam. This event tree is a good starting point in the development of the actual event tree for the dam being analyzed. The four components of the event tree are annual loading probability, probability of unsatisfactory performance, performance level, and consequences. Consequences may include economic and environmental impacts and loss of life. For this example, only economic impacts are considered. Risk is the combination of the probabilities of occurrence of the events and the adverse consequences of the events. Examining the event tree in Figure 2, the risk for each branch of the event tree is the product of the four components making up that branch (annual loading probability, probability of unsatisfactory performance, performance level, and consequences). The annual economic risk for the event tree is the sum of the risks for each branch. The annual economic risk calculated using the event tree is then used in the economical analysis for a Major Rehabilitation Report to determine the economical feasibility of the recommended rehabilitation measures.

b. Independent Failure Mode Versus Sequential Failure Mode Event Trees. A separate event tree should be created for each independent failure mode. This means a separate event tree is required for each failure mode that is not conditionally dependent on another failure mode. For example, where a slope stability failure mode is independent of a detrimental seepage failure mode, two separate event trees would be developed. However, if a slope stability failure caused detrimental seepage to develop, then only one event tree is used showing the sequential relationship of the failure modes. Figure I-4 in Appendix I shows an event tree using sequential failure modes. The sequential events are defined in terms of conditional probabilities.

c. Conditional Probabilities. Moving across the event tree shown in Figure 2, the components of the event tree represent the probability of a pool elevation occurring (P(Pool Elev)), the probability of unsatisfactory performance given that this pool elevation has occurred (P(u|Pool Elev)), the probability of a performance level occurring given that there has been unsatisfactory performance (P(Perf Level|u)), and the cost incurred if that performance level has occurred ($|P(Perf Level)).The last column in the event tree is the risk associated with each branch of the event tree. These risks are summed to get the annual economic risk for the event tree. All the probabilities under the annual loading probability category of the event tree must sum to one. The events in the second and third components of the event tree seen in Figure 2 represent conditional probabilities. Conditional probability is the probability of an event occurring given that another event has occurred. For example, say that there is a 10 percent probability of a slide given that a certain water level has occurred. Moving across the components of the event tree, values for probabilities and costs are multiplied together giving expected risk as the result. The risks in the final column of the event tree are summed to give the annual economic risk of unsatisfactory performance of the dam. Equation 2 represents the calculation of the annual economic risk.

$$\text{Annual Economic Risk} = \sum P(\text{Pool Elev}) \times P(u|\text{Pool Elev}) \times P(\text{Perf Level}|u) \times \$|P(\text{Perf Level}) \quad (2)$$

d. Annual Loading Probability.

(1) The main loading condition that a dam experiences during its life is caused by the water that is impounded by the dam. The water level acting on the dam varies daily. The probability of different water levels acting on the dam can be obtained from a pool elevation frequency curve. The pool elevation frequency curve represents the annual probability of a certain water level acting on the dam being equaled or exceeded. The pool elevation frequency curve that will be used for illustration purposes is shown in Figure 3. The water level on the dam is plotted on the y-axis. The annual probability that the water level equals or exceeds that elevation is plotted on the x-axis. Therefore, probabilities of unsatisfactory performance calculated from the pool elevation frequency curves are annual probabilities of unsatisfactory performance.

(2) The annual loading probability on the dam is obtained from the pool elevation frequency curve. This annual loading probability is represented in the event tree by the probability that the pool will be at various elevation ranges P(Pool Elev). The pool elevation frequency curve gives the annual probability that the water level will equal or exceed an elevation. But for the reliability analysis, the probability that the water level will be at a certain elevation is needed. The simplest method to obtain the probabilities that the water levels are at certain elevations is to represent each elevation as a range. For the example shown in Table 1, pool elevations 440.2, 435.2, 432.5, 429.4, 424.5, 420.7, and 416 feet are selected because they correspond to the return period of a 500, 100, 50, 25, 10, 5, and 2-year event, respectively. Elevations approximately midway on either side of each pool elevation are entered into the table except for the highest increment and lowest increment. Elevation 442.5 was chosen for the highest increment, as that is the top of the dam. Elevation 400 was chosen for the lowest increment, as that was the lowest point on the pool elevation frequency curve. For each elevation increment, the probability that the pool elevation equals or exceeds that elevation (P(exceedence)) is selected from the pool elevation frequency curve, converted from a percentage to a decimal, and entered into the table. The annual probability that the pool is at a certain elevation (P(Pool Elev)) represented by a range is the difference between the P(exceedence) at the top and bottom of the increment for that range.

TABLE 1
PROBABILITY OF POOL LEVELS OCCURRING

Pool Elev	Elevation Increment	P(exceedence)	P(Pool Elev)
	442.5	0	
440.2			0.005
	437.5	0.005	
435.2			0.011
	433.5	0.016	
432.5			0.012
	431.0	0.028	
429.4			0.032
	427.0	0.060	
424.5			0.08
	422.5	0.14	
420.7			0.2
	418.0	0.34	
416.0			0.66
	400.0	1.00	
		Σ	1.000

The probability that the pool is at Elevation 429.4 is determined using Equation 3.

$$P(\text{Pool Elev})_{429.4} = P(\text{equals or exceeds})_{427} - P(\text{equals or exceeds})_{431} \quad (3)$$

Applying Equation 3, the probability that the pool is at elevation 429.4 is determined to be 0.032:

$$P(\text{Pool Elev})_{429.4} = 0.06 - 0.028 = 0.032$$

The probability that the pool is at the other elevations in the table is determined in a similar manner.

The probability that the pool is at Elevation 429.4 is really the probability that the pool is in a range between elevations 431 and 427. If using this size range is not accurate enough, then more pool elevations should be analyzed and the range size reduced.

e. Probability of Unsatisfactory Performance. The next category of the event tree is the probability of unsatisfactory performance. Four methods are available to obtain the probability of unsatisfactory performance: hazard function, reliability index, expert elicitation, and historical frequency of occurrence. The probability of unsatisfactory performance must be determined for each pool level in Table 1 using the most appropriate of the above four methods. The probability of unsatisfactory performance can be determined using the hazard function or the reliability index method for failure modes for which there is an analytical method of analysis. For failure modes for which there is no analytical method of analysis, the probability of unsatisfactory performance can be obtained by either expert elicitation or historical frequency of occurrence for each pool elevation.

(1) Hazard Function. The hazard function h(t) is defined as the conditional probability of an event occurrence in the time interval $t+\Delta t$, given that no event has occurred prior to time t. To determine a hazard function using analytical methods, parameters must exist that vary with time in a known manner. Examples of parameters that vary with time are corrosion of a steel member, fatigue of a steel member due to cyclic loading, and scour around a foundation member. A hazard function can be calculated using a Monte Carlo analysis. This is most easily done for analyses that can be accomplished using a spreadsheet analysis in conjunction with Monte Carlo simulation software @RISK. A Monte Carlo simulation requires the analysis to be run thousands of times varying the random variables. The number of unsatisfactory performance events is counted in each year time increment allowing the hazard function to be calculated. , While some geotechnical parameters may be time dependent, only limited information exists on the mathematical functions defining how they vary with time. Thus, it is difficult to calculate a hazard function for use in Monte Carlo simulation. Therefore, a hazard function analysis is not typically done for seepage and slope stability problems using Monte Carlo simulation. Table 2 gives a discussion of various geotechnical parameters and information on how they vary with time. For most geotechnical parameters that vary with time, historic data is needed to define the time function. For this reason a historic frequency of occurrence model may be a better choice for the reliability analysis if there is historic data defining unsatisfactory performance.

TABLE 2
VARIATION OF GEOTECHNICAL PARAMETERS WITH TIME

GEOTECHNICAL PARAMETER	DISCUSSION
Shear Strength	The shear strength of sand is fairly constant with time. The shear strength of clay increases with time in a predictable fashion but that would give an increasing hazard function and would be of little use in justifying major rehabilitation of projects.
Settlement	The major amount of settlement of sands occurs fairly rapidly; however, we do know that additional settlement occurs with time that is somewhat predictable. Clay settles by a predictable rate over time; therefore, settlement of a structure on clay would be a perfect model for a hazard function analysis.
Scour/Erosion	Since some scour events are time dependent, a scour model would work well with a hazard function analysis. The only problem is that data is needed to predict how the scour will occur. Therefore, the scour event is predicted by historic data and the historic data could be used in a Monte Carlo simulation to predict the hazard function for a sliding model. Scour is predictable if the material being scoured is uniform throughout. But if there is layered material or discontinuous zones, it would be very difficult to predict

Scour/Erosion (continued)	the rate of scour with time. This method would work with erodable rock versus a known scouring agent, but not scour of a sand foundation at a bridge pier.
Permeability/Piping/and Underseepage	Permeability can change with time especially in the area of a known underseepage problem when foundation material is being removed with time. The problem with using a Monte Carlo simulation to analyze the underseepage problem is that no one knows how the permeability changes with time.
Relief Well Flow	Relief well flow is definitely time dependent as the screen of the well and the filter become plugged over time due to precipitation of chemical compounds or biological fouling. However the rate of reduced flow changes drastically depend on the location of the relief well. There is no mathematic model available to predict the change of relief well flow with time to use in a Monte Carlo simulation to calculate a hazard function.
Deformation of Pile Foundation	Lateral deformation of a pile foundation increases with the number of cycles of loading applied, which can be related to time. Data exists on the additional lateral deformation of a pile foundation that occurs with time due to cyclic loading and a time model can be developed.
Loading	Loading certainly can vary with time and for certain types of loading a time function can be developed. This would be very problem specific and would not apply to a large group of slope stability or seepage problems which is the topic of this ETL

(2) Reliability Index. The reliability index method is a practical and appropriate method for reliability analysis of geotechnical problems. Guidance for geotechnical reliability analysis is given in ETL 1110-2-547. When performing reliability analyses, the same analytical methods are used that are used in traditional geotechnical engineering. The only difference is in how the variables in the problem are represented. In reliability analysis, there are two types of variables: deterministic and random. In traditional geotechnical engineering, all variables are treated as deterministic. Deterministic variables are represented by a single number, implying the value of that variable is known exactly. A deterministic variable could be the unit weight of water or measured dimensions. Random variables are represented by a probability density function because their exact value is not known. Random variables could be shear strength, permeability, and earth pressure. The Corps of Engineers uses what is called a first order second moment method of analysis for the reliability index method. This means that only the first two moments (mean and variance) are used to represent the probability density function in the reliability analysis and all higher order terms are neglected in the Taylor Series expansion used to estimate the mean and variance of the performance function (natural log of the factor of safety). This first order second moment method of analysis greatly simplifies the reliability analysis procedure. This simplified reliability analysis is used to determine the reliability index beta (β). β is the number of standard deviations by which the expected value of the factor of safety is away from

the unsatisfactory performance condition, or the factor of safety equaling one. The larger the value of β, the safer the embankment dam. The smaller the value of β, the closer the embankment dam is to unsatisfactory performance. The value of beta can be used to calculate the probability of unsatisfactory performance. The reliability index provides a measure of the likelihood of unsatisfactory performance for a given performance mode and loading condition. If a distribution on P(u) is assumed, it can be used to calculate the probability of unsatisfactory performance for the considered loading condition. It does not, however, have a time basis, or directly provide the annual probability of failure for the considered mode.

Life-cycle economic simulations performed by the Corps of Engineers require annual probabilities of various unsatisfactory performance events. These probabilities are defined by hazard functions, which may be constant or vary with time. The hazard function or hazard rate h(t) is the conditional probability of failure in the current year, given that there was no failure in the previous years. It can be expressed as the ratio of the probability of failure distribution function at that time to one minus the cumulative failure probability at that time. A general discussion of the application of the hazard function in geotechnical engineering is presented in paragraph 2.e.(1).

(3) Expert Elicitation. Expert elicitation is a systematic process of posing questions to a panel of experts to get their knowledgeable opinion on the condition of the embankment dam so that the probability of unsatisfactory performance can be assessed. After the initial round of questions has been completed, the expert opinions are documented. The experts then explain their the initial documented opinions. Some experts may want to adjust their initial opinions based on this discussion. The probabilities are based on the panel's revised opinions. The expert elicitation process must be thoroughly documented. Further discussion is presented in Appendix E.

(4) Historical Frequency of Occurrence. Historical frequency of occurrence is the collection of data at the dam being investigated or at similar dams from which the probability of unsatisfactory performance is determined. The one disadvantage of the method is that it is usually very difficult to obtain this data. Sometimes the historical data has been collected and reduced by others, such as in the University of New South Wales procedure (Foster, 1998), which is used for the analysis of piping through embankment dams. A site-specific analysis can be performed using frequency-based methods fit to historical events. Ways to reduce the historical events into a usable form is through the use of common data distribution methods, such as, the Weibull, Exponential, or Poisson data distribution methods. Note, using Weibull, Exponential, or Poisson distribution methods to reduce historical data gives a hazard function. Appendix J gives an example for developing a historic data model using the Weibull and the Exponential distribution. An example using the University of New South Wales procedure is given in Appendix H.

f. Performance Level.

(1) After the probability of unsatisfactory performance is calculated, the engineer must assess what effect this would have on the embankment dam. Remember that what has been calculated is the probability of unsatisfactory performance not the probability of failure. So the

engineer must determine exactly what unsatisfactory performance means. Typically the degree of unsatisfactory performance can be represented by three performance levels; however, as many performance levels as needed to describe the problem being analyzed can be used. When using three performance levels the following can be used:

(a) Low Impact. The low impact performance level requires some increased maintenance effort on the embankment dam.

(b) Extreme Measures. The extreme measures performance level requires a major effort and/or expenditures of funds to prevent uncontrolled release of the pool, such as, pool restriction and extensive emergency construction measures to prevent failure of the dam.

(c) Catastrophic Failure. The catastrophic failure performance level is breaching of the dam and release of pool with or without warning with subsequent economic and environmental impact and loss of life.

(2) These are just three examples of what unsatisfactory performance can mean. There can be more than or less than these three outcomes of the unsatisfactory performance. These outcomes are referred to as the performance levels of the embankment dam assuming that the dam performs unsatisfactorily. The probability of each of these performance levels occurring must be determined by the engineer. The sum of these performance level probabilities must add to one. The engineer should determine performance level probabilities based on experience, historical data, or other expert opinion.

g. Consequences.

(1) The consequences are determined using a multi-disciplinary team of engineers, planners, economists, environmental resource specialists, and operating project managers for each performance level. The consequences (economic and environmental impacts both upstream and downstream and loss of life) must be determined for each performance level for each range of pool elevations where unsatisfactory performance will occur. This does not necessarily mean uncontrolled loss of the pool has to occur before unacceptable consequences would be incurred. Extreme consequences (economic and environmental impact) could occur without catastrophic loss of the pool just by restricting the pool elevation to prevent embankment dam failure. Impacts could be realized due to reduction in downstream flood protection (not being able to store inflow and causing flooding downstream), loss of hydropower benefits, damage to environmental habitat units, and impacts on operations of upstream projects. There would be system impacts if the project is part of a coordinated flood control, navigation, and hydropower system.

(2) Consequences for catastrophic failure should be determined based on the increase in damages over the base discharge flow at the time of dam breaching. The consequences should be determined for each representative pool elevation used in the event tree. The consequences should include loss of life; the cost of repair of the dam; damage to upstream and downstream property; environmental damage and repair; loss of benefits (hydropower, flood control,

navigation, recreation, water supply); cost of a monitoring program; and delays to transportation systems.

h. Annual Economic Risk.

(1) Once the impacts are determined, the risk portion of the event tree can be calculated. This is accomplished using Equation 2 and the event tree. The probabilities and costs are multiplied for the annual loading frequency, the probability of unsatisfactory performance, the performance level, and the consequences for each branch of the event tree. These values, which are in terms of dollars, are the risks. All of the individual branch risks are summed to give the annual economic risk for the without–project condition. This means the project as it is now without any repairs being made to the dam.

(2) This whole process must be repeated to get the annual economic risk for the with-project condition. This means the project after the repairs have been made to the dam. The main changes in the event tree between the without-project condition and the with-project condition will occur in the probability of unsatisfactory performance category of the event tree. The probabilities of unsatisfactory performance will be much lower after the embankment dam has been repaired. A simplifying assumption that is used is that when rehabilitations are completed (With Project), they are made to bring the structure up to current criteria. Therefore, there should be an insignificant chance of having a reliability problem in the future. In addition, if there is a problem it would most likely be towards the end of the study period where the present worth aspects of the economic analysis would make the effect insignificant to the overall analysis.

(3) The annual benefits portion of the economic analysis is then calculated using Equation 4.

$$\text{Benefits} = \text{Annual Economic Risk(Without Project)} - \text{Annual Economic Risk(With Project)} \quad (4)$$

These benefits are used by economists to obtain the benefits-cost ratio.

i. Alternate Method of Calculating Benefits. The event tree would not change since the same pool elevations would be used, but instead of using the probability that the water level is at an elevation, the pool elevation frequency curve probability of exceedence would be used. This method of calculating the benefits is illustrated in Figure 4. Each pool elevation of interest is represented by a diamond on the curve. The exceedence probabilities are plotted on the x-axis. The weighted damages, which are calculated using Equation 5 for each pool elevation of interest, are plotted on the y-axis.

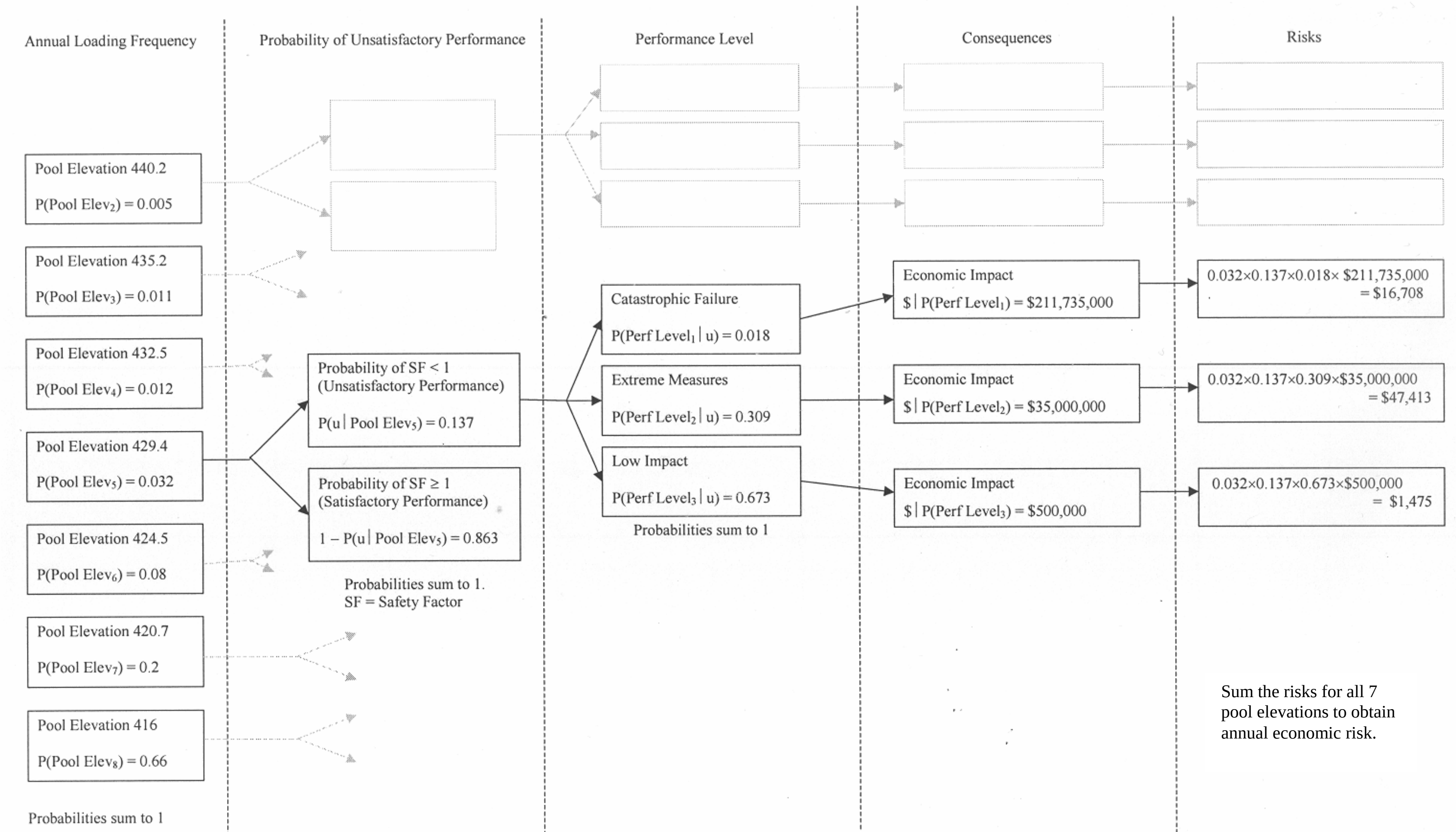
$$\text{Weighted Damages} = \sum P(u|\text{Pool Elev}) \times P(\text{Perf Level}|u) \times \$|P(\text{Perf Level}) \quad (5)$$

Note that Equation 5 does not contain the probability of a pool elevation occurring, as does Equation 2. The probability of a pool elevation occurring is taken into account in the area under the curve calculation, i.e., the weighted damages are multiplied by the exceedence probability of the pool.

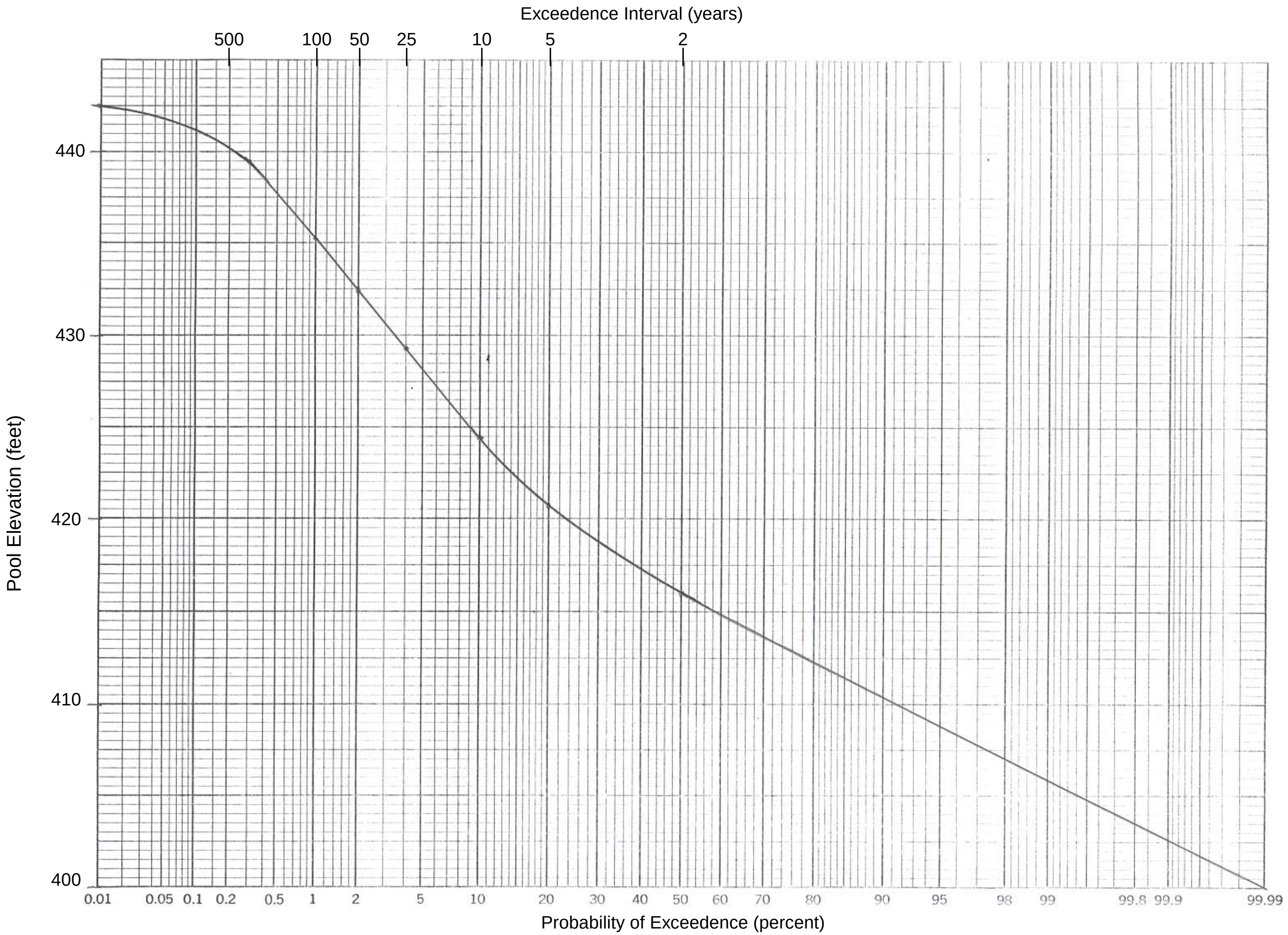
Using Equation 5, the weighted damages for pool Elevation 429.4 are calculated as follows using the data in the event tree, Figure 2:

Catastrophic Failure	$0.137 \times 0.018 \times \$211,735,000$	=	\$522,139
Extreme Measures	$0.137 \times 0.309 \times \$35,000,000$	=	1,481,655
Low Impact	$0.137 \times 0.673 \times \$500,000$	=	46,101
Weighted damages at Elevation 429.4		=	\$2,049,894

From Figure 3, Elevation 429.4 corresponds to a probability of exceedence of 0.04. The weighted damages of \$2,049,894 corresponding to a probability of exceedence of 0.04 or a 25-year return period are shown in Figure 4. This calculation gives one data point on the curve. Other pertinent pool elevations would be used to compute the rest of the data points on the curve in Figure 4. The area under the curve in Figure 4 is the annual economic risk. A curve similar to Figure 4 would be developed for both the without and with project conditions. The difference between the areas under the two curves gives the annual benefits.

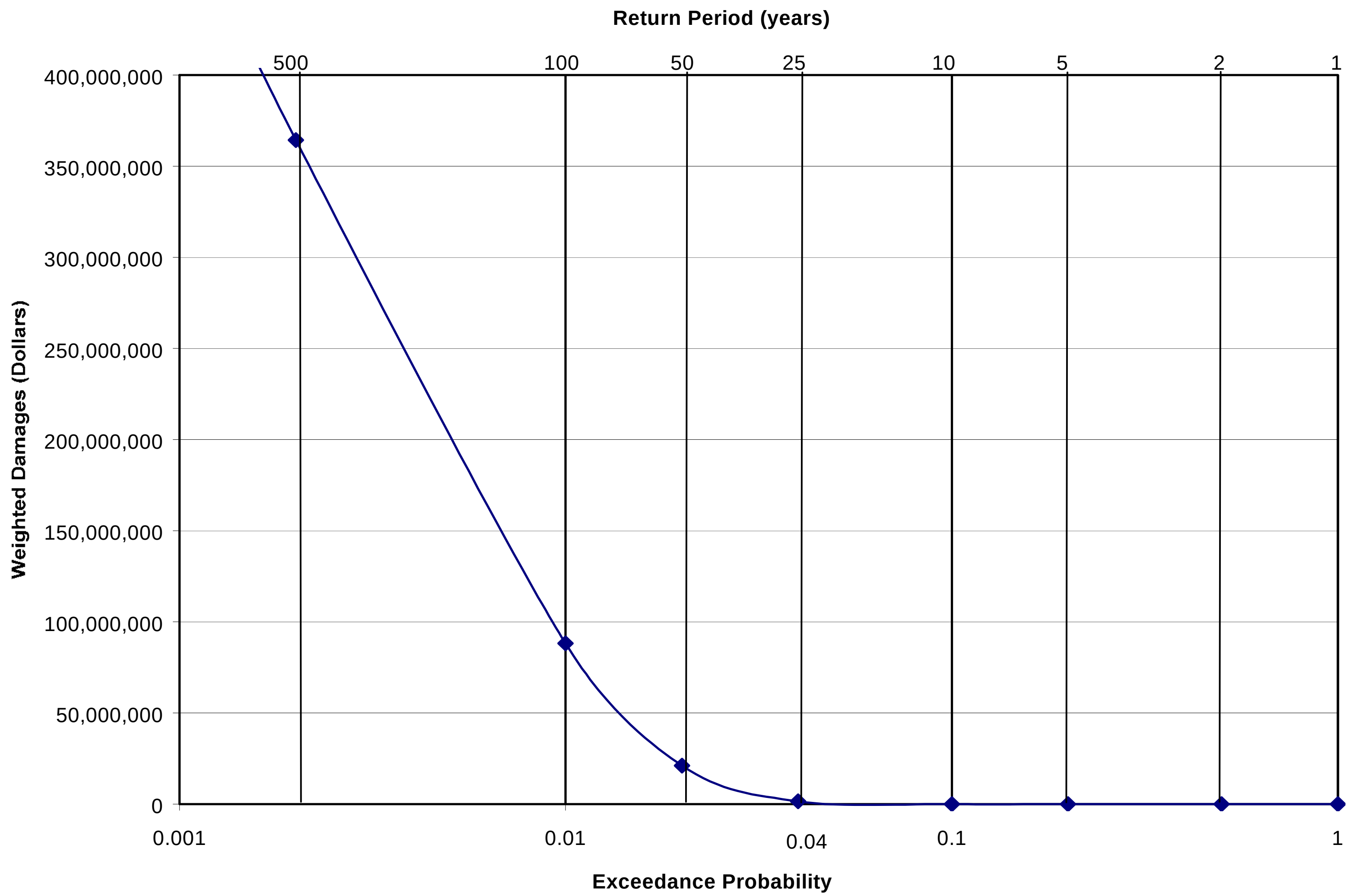


Event Tree Example
Figure 2



Pool Elevation Frequency Curve

Figure 3



AVERAGE ANNUAL NED COST

Figure 4

3. Seepage Risk Analysis

a. Problem Definition.

(1) Seepage, through an embankment and foundation of a dam, can, if not properly controlled, cause failure of the dam. Direct observation of unsatisfactory performance such as excessive piezometric levels, sand boils, turbidity in seepage discharge, increases in seepage discharge over time, or changes in piezometric head for similar past pool elevations are all indicators of potentially serious seepage conditions. These conditions may be indicative of detrimental effects of seepage leading to piping and eventual breaching of an embankment dam or the foundation of a dam.

(2) Currently there are identified two different piping conditions requiring different methods of analysis: 1) piping directly related to pool elevation which can be analyzed using piezometric or seepage flow data, and 2) piping related to a complex seepage condition within the dam or its foundation that cannot be evaluated solely by analytical methods. Analysis of the first condition usually entails extrapolating observed piezometric data at various lower pool levels to predict if failure or unsatisfactory performance, defined as excessive seepage exit gradients, will exist at higher pool levels. This relationship is presented in Figure 5, Estimated Uplift vs. Pool Elevation - Performance Parameter Plot at Toe of a Dam. The second condition relies on the analysis of historical frequency of occurrence data and/or expert elicitation to determine the probability of failure by seepage and piping using either frequency-based methods to fit historical events and/or subjectively determine probability values based on expert elicitation. These generalized approaches are summarized in Figure 6.

b. Project Site Characterization. Site characterization consists of identifying those factors that are significant in the evaluation of through seepage, under seepage, and piping potential. Typical factors would include embankment zoning and material classifications, compliance with current filter and drain criteria, foundation materials and stratigraphy, seepage cutoff and control features, and performance history. The University of New South Wales (UNSW) method for conducting preliminary assessment of the probability of failure by piping provides a good example of factors considered in the evaluation process.

c. Define Mode(s) of Failure for the Site. As stated before, uncontrolled seepage through the embankment and foundation system of a dam can cause failure of a dam. Uncontrolled seepage through the embankment can lead to piping of the embankment with subsequent erosion of the embankment through the “pipe” with eventual breaching by the pipe at the upstream face allowing breaching of the embankment. Seepage through the foundation can cause excess uplift at the downstream toe of dam and “heave” the foundation material at the toe, which can lead to piping of the granular foundation material. Uncontrolled seepage through the embankment into the foundation can lead to movement of the embankment material into the foundation material creating a pipe or cavity in the embankment which will eventually collapse and cause loss of free board or a breach through the embankment. Internal erosion or movement of fine-grained material through the embankment can cause failure of the embankment (Talbot, 1985).

d. Methods to Determine Conditional Probability of Unsatisfactory Performance.

(1) Reliability Index Method. Where piping/seepage performance is correlated with pool level/elevation the First Order Second Moment Reliability Index method, presented in ETL 1110-2-547, can be used to determine the conditional probability of unsatisfactory performance. See Appendix G of this ETL for an example on how this method was used to determine the conditional probability of unsatisfactory performance.

(2) Historical Frequency of Occurrence and/or Expert Elicitation Methods. Factors leading to piping often may not be addressed by traditional seepage analysis. This is particularly true when dealing with the movement of fine-grained material through the embankment or foundation of a dam. Thus reliability analysis methods based on historical frequency of occurrence and/or expert elicitation are appropriate to obtain the average annual probability of failure by seepage and piping. One method based on historical frequency of occurrence is the UNSW method and another is the use of expert elicitation using available site specific information.

(a) UNSW Method. The UNSW method gives the probability of failure and per annum failure rate without any direct relationship to pool elevation. See Appendix H for an example using a historical data model. It is recommended that the normal operating pool (up to spillway crest for ungated spillways) be used to determine consequences of breaching failure for this method. The per annum failure rate should be directly multiplied by the consequences related to the normal operating pool elevation to obtain the annual economic risk.

(b) Expert Elicitation. A site-specific analysis can be performed using a combination of frequency-based methods fit to historical events and subjectively determined probability values based on expert elicitation. When using expert elicitation one needs to rigorously document the process used and provide that with the report. Appendix E contains detailed guidance on the use of expert elicitation in geological and geotechnical engineering. An example of this method used for Walter F. George Dam is presented in Appendix F. Walter F. George Dam, Mobile District, experienced seepage through the solutioned limestone foundation, and uncontrolled seepage events have occurred at seemingly random locations on random occasions unrelated to pool level. Having no situation readily amenable to analytical modeling, the risk assessment was performed using a combination of frequency-based reliability methods fit to historical events and subjectively determined probability values based on expert elicitation. Given the set of historical events, annual probabilities of new events were taken to be increasing with time due to the continued solutioning of the limestone. The expert panel estimated probability values for future seeps occurring at various locations, for locating the source of the seep in sufficient time, for being able to repair the seeps given that they are located, and for various structural consequences of uncontrolled seepage.

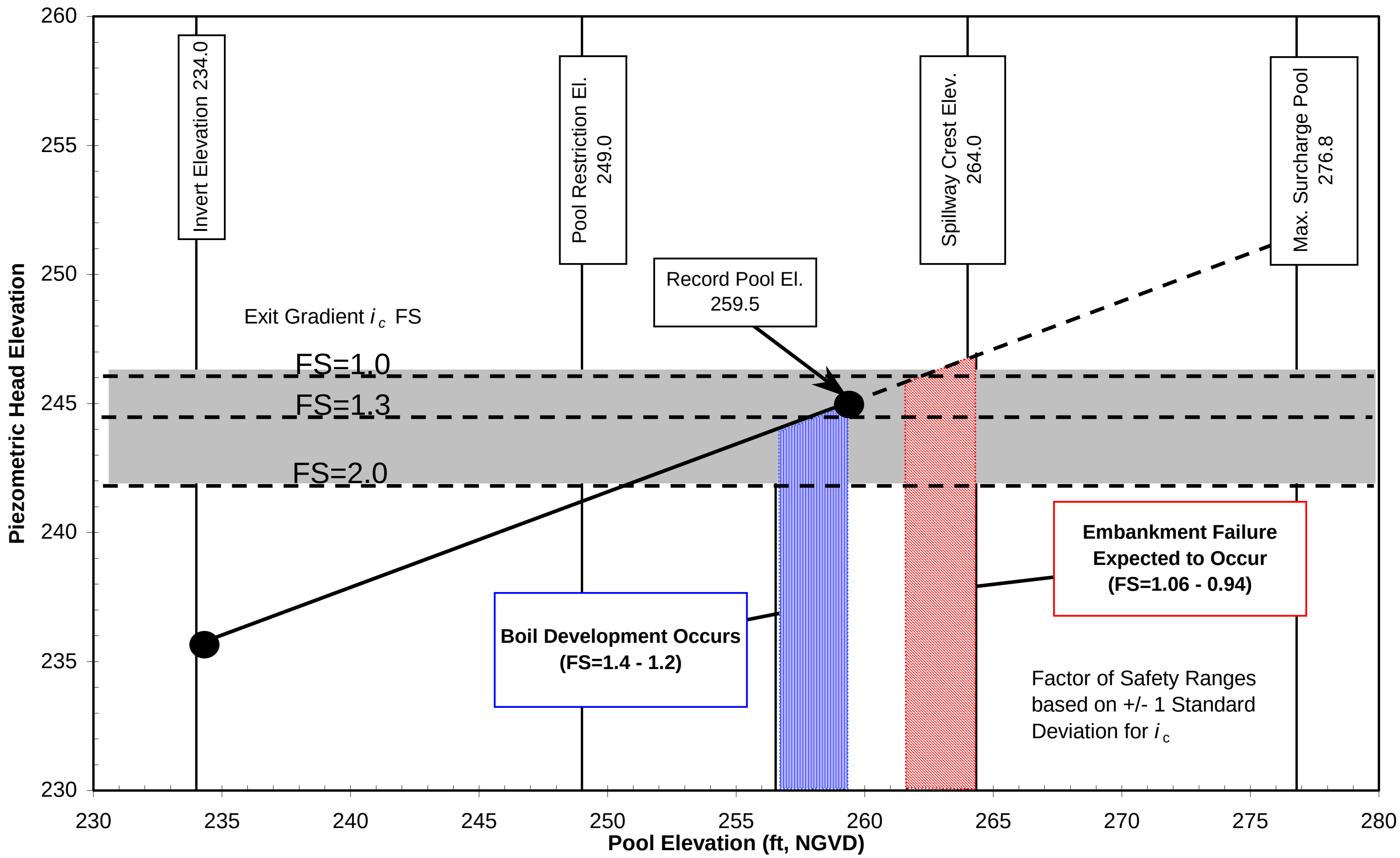


Figure 5. Estimated Uplift vs. Pool Elevation - Performance Parameter Plot for Conditions at Toe of a Dam

Analytical Based Method	**Expert Elicitation Method and/or Historical Frequency of Occurrence Method**
Piping/Seepage performance, i.e. exit gradient, can be directly related to pool elevation.	Piping/Seepage performance related to a change in conditions over time.
(Annual Pool Level Probability) **X** **(Conditional Probability of Unsatisfactory Performance)** **X** **(Performance Level)** **X** **(Consequences)** **=** **Annual Risk for Each Pool Level**	
Sum Annual Risk for Each Load to Obtain the Annual Economic Risk for Seepage and Piping Failure.	

Figure 6. Generalized Concept of the Different Piping/Seepage Risk Assessment Approaches.

4. Slope Stability Risk Analysis

a. Problem Definition.

A risk analysis or risk assessment of the slope stability of a project might be deemed appropriate due to changes in performance of the slope, changes in design criteria, or changes to the loading conditions.

(1) For example, a failure in a specific dam would suggest that all dams of similar design and construction are suspect and would require reanalysis and possible remediation.

(2) Changes in pore pressure or piezometric surface are indicators that potential stability problems might be developing, thus necessitating reanalysis.

(3) When measured deformations in the embankment and or foundation indicate that the embankment is not performing adequately, reanalysis and remediation may be required.

(4) Design criteria changes may indicate, after reanalysis, that remediation is required.

(5) When there is a change in the reservoir loading conditions, normally a change in the permanent pool, an increase in elevation of the normal pool range or operating range, reanalysis and possibly remediation will be required.

b. Project Site Characterization.

Site characterization consists of identifying those factors that are significant in the evaluation of slope stability.

(1) Typical factors would include embankment geometry, zoning and material properties, foundation geologic profile and material properties, pore pressures and piezometric profiles, and as-built conditions.

(2) Historical performance of the embankment or slope can provide evidence or indications of zones of movement, areas of high pore pressure, relationship of movement with pool elevation, rain fall, or groundwater levels.

(3) To reduce the level of uncertainty in the analysis the range of values for the significant input parameters need to be identified as well as possible and this may entail additional field exploration and laboratory testing.

c. Loading Conditions, Mode of Failure and Implications.

(1) As an existing project is being analyzed, there are two loading conditions that may be considered - the Steady State and Rapid Drawdown loading conditions.

(2) Shallow vs. Deep Seated Failure.

(a) Shallow shear failure, in which only a small shallow portion of the embankment moves, is designated as a surface slough. Such failures result when the slope is steeper than the soil shear strength can resist or when the seepage gradients are large enough to cause slope movement (for example, sudden drawdown loading condition). These failures are considered a maintenance problem and usually do not affect the structural capability of the embankment. However, if such failures are not repaired they can become progressively larger and may then represent a threat to the embankment safety. Implications are significant in instances where the shallow slides impact the intake tower or block the intake tower or spillway channel entrance.

(b) Deep shear failure, which involves the majority of the embankment cross-section or the embankment and the foundation, can reduce the crest elevation of the embankment thus increasing the potential for overtopping of the dam leading to uncontrolled release of the pool. Deep shear failure with large lateral movement can compromise the seepage control system of a zoned dam or dams with seepage control systems installed in the foundation at the downstream toe of an embankment. Where there is movement in the embankment and foundation in the area of the regulating outlet works the embankment tie in with the outlet works can be compromised and the outlet works could be damaged, which could lead to piping failure of the embankment along or into the outlet works conduit.

(3) Concrete Structure Sliding on Shear Surface in Rock. Concrete structures founded on weak rock or rock with layers of weak material may experience shear failure and the movement can cause loss of the pool. This movement could compromise an embankment tie-in with the concrete structure or an abutment contact with the concrete structure. Guidance for analysis of gravity structures on rock foundations can be found in Corps of Engineers Technical Report GL-83-13, Design of Gravity Dams on Rock Foundations: Sliding Stability Assessment by Limit Equilibrium and Selection of Shear Strength Parameters, October 1983.

(4) Process to Define Critical Shear Surface or Zone.

(a) For projects with observed movement or deformation there will normally be adequate data to locate the shear surface within reasonable bounds in the embankment and foundation. Use this surface to determine the probability of unsatisfactory performance using the reliability index method.

(b) For those projects where there is no clearly defined zone of movement a general discussion on determining critical slip surfaces is given on pages A-16 to A-17 of ETL 1110-2-556. However, we recommend that engineering judgment be used to guide the selection of the slip surfaces used in the reliability analysis.

<u>1</u> Search for the slip surface corresponding to the conventional critical Factor of Safety slip surface and determine the reliability index for that slip surface.

<u>2</u> Additionally, identify slip surfaces that would compromise critical features of the dam and determine the reliability index for those slip surfaces.

3 This set of slip surfaces would then be used to guide the selection of slip surfaces to be used in the risk assessment.

4 It should be noted that the slip surface that gives the minimum factor of safety does not necessarily correspond to the slip surface that gives the minimum reliability index (β). In fact the minimum β value may differ significantly from that calculated for the minimum factor of safety slip surface. Therefore it is imperative that the slip surface that gives the minimum β value be found. Presently there are no slope stability programs available that search for the slip surface that gives the minimum β value. An empirical algorithm to search for the critical probabilistic surface was published by Hassan and Wolff (1999).

(5) Two-Dimensional Versus Three-Dimensional Analysis Methods.

(a) For the vast majority of cases two dimensional analysis methods are satisfactory.

(b) Use of three-dimensional analysis methods is recommended when there is significant evidence indicating that the failure surface is constrained in such a manner that the plain strain assumption used in two-dimensional analysis is not valid, as in the case in which the potential failure surface is clearly constrained by physical boundaries.

(c) Typically this is only for translational failure masses where use of a sliding block with vertical sides is recommended, as it is simple to model and verify. Side forces are calculated as a function of at-rest earth pressure.

d. Conditional Probability Of Unsatisfactory Performance.

(1) Reliability Index. The conditional probability of unsatisfactory performance is determined by the reliability index method presented in ETL 1110-2-556, pages A-10 and A-11. The use of a well-documented and tested slope stability analysis program is recommended to compute the factors of safety used in the reliability analysis. Example problems are included in Appendices D and I to illustrate the use of the reliability index method as it relates to slope stability analysis.

(2) Reliability Index Factor versus Design Factor of Safety for Slope Stability.

(a) Comparisons of traditional factor of safety design methods to reliability index approaches generally concentrate on the ability of the probability-based methods to deal with reliability in a more comprehensive way than do the factor of safety methods. This view is stated well by Christian (1996):

> “The reliability index provided a better indication of how close the slope is to failure than does the factor of safety alone. This is because it incorporates more information – to wit, information on the uncertainty in the values of the factor of safety. Slopes with large values of β are farther from failure than slopes with small values of β regardless of the value of the best estimate of the factor of safety.”

(b) However, the factor of safety entails more than a measure of reliability. Mechanically, the factor of safety is the ratio of forces that can be mobilized as strength to the forces tending to cause slope movement. Harr (1987) has described the factor of safety in terms of capacity and demand; for slope stability, the strength represents the capacity of the slope while the driving forces represent the demand. A factor of safety of one signifies that demand is just balanced by capacity and the system is at a limit state. If demand exceeds capacity, the slope is in a state of failure. A well-engineered structure requires a reserve in capacity implying a factor of safety greater than one. The particular choice of the factor of safety depends on both the type of loading and the type of structure. Recommended values are generally based on experience of structures that have, or have not, experienced satisfactory performance.

(c) The difficulty with incorporating the factor of safety into a reliability analysis is twofold. First, the experience with traditional factor of safety-based design method is based on a sequence of practices involving site investigation, laboratory technique, and data interpretation where each part of the sequence entails some level of conservatism. Second, the factor of safety conveys more than a level of reliability because failure of an earth structure is not a discrete event. The practical failure of a capacity-demand system generally occurs prior to extracting full capacity. In the case of an embankment slope, the capacity (strength) is extracted at the cost of deformation, which might have ramifications that would be considered unsatisfactory performance even before the factor of safety falls to one. Therefore, the reliability index method separates the issue of uncertainty from that of performance. The factor of safety is treated as a random variable through a procedure where the uncertainty of the computation process can be dealt with objectively. The question of acceptable factor of safety is dealt with through the performance level factor.

(3) Expected Value Versus the One Third/Two Thirds Rule.

(a) Corps practice has been to select the design strength such that it is less that two-thirds of measured strength values. For reliability analysis, this criterion is supplanted by the reliability index method, which is a process of quantifying the probability distribution of strength values thereby determining the probability distribution of the factor of safety. When the shear strength parameters are selected in accordance with the Corps of Engineers one third/two thirds rule, instead of the expected value, the β is inappropriately low. In its proper application, each step in computing the reliability index should be performed using an objective estimate of the expected value and variance.

(b) In reliability analysis, emphasis is on comparing the best estimate of actual prevailing or predicted conditions to those, which correspond to failure or unsatisfactory performance. Hence, it is essential that expected values be assigned in an unbiased manner that does not introduce additional conservatism, so that the reliability index β remains an accurate comparative measure of reliability.

(4) Tables of Coefficient of Variation. Often there is not adequate data to determine the coefficient of variation. When this is the case, it is recommended that typical values for the coefficient of variation be used. Typical values of tabulated data are presented in Table D-2 of Appendix D and Table 1, Appendix B, ETL 1110-2-556.

e. Performance Level.

(1) The traditional selection of factor of safety is supplanted by assigning a performance level factor. Given a low factor of safety, a number of levels of performance can be envisioned for that mechanism, with each having their own probability. For given loading cases and degree of performance a performance level value needs to be assigned for those values of the factor of safety where unsatisfactory performance is determined to occur. A factor of safety is tied to a mechanism (trial shear surface). It is generally implied that the mechanism having the lowest factor of safety approximates the geometry of the failure that would actually emerge. Several non-minimal surfaces might be considered unreliable because a high probability of failure is associated with both low factor of safety and level of uncertainty of either capacity or demand. Also, the minimal slip surface is often associated with shallow slips that are of limited consequence.

(2) The use of a performance level factor is incorporated into the risk calculations by assigning probabilities to the various possible outcomes associated with each factor of safety. The level of performance should take into account the susceptibility of the embankment to catastrophic failure by the various failure modes determined for the embankment under study. This may require an over topping analysis or a seepage piping and erosion analysis. Alternatively, catastrophic failure does not necessarily occur when an analysis shows the slope has reached the limit state defined by FS=1. Usually, levels of performance are associated with the amount of movement associated with failure. The probability of the discrete event of the factor of safety equaling one must be assessed in view of the reality that movement after failure is limited. Similarly, there might be considerable movement prior to reaching a factor of safety of one. Thus, an unsatisfactory performance level might be probable even if a full failure state is not reached, especially for slopes that have documented distress at levels of FS > 1. The general assumption of a slope stability calculation is that strength is mobilized along the entire potential sliding surface. The reality is that often movement is observed in an embankment or slope indicating that at least part of the slope has reached a limit state. In the particular case where much of the strength capacity is derived from passive resistance, significant movement might occur in the active block before FS=1 is obtained. This movement might or might not be indicative of poor performance. Typically some major change in reservoir operations can be enacted to reduce the load on the slope allowing some remedial action to be taken, which incurs a cost and impacts project purposes.

(3) The value for the performance level can be determined using case histories, expert elicitation, and engineering experience. Several options are available:

(a) Use the recommended factors of safety as the limit between satisfactory and unsatisfactory performance. Because conservative strength values were part of traditional design, the recommended safety factors were experience-based measures of satisfactory performance. (In this case the computation of performance level must be modified for FS>1 as described in Appendix D.)

(b) One alternative is a static deformation analysis. This needs to consider the available free board and the estimated total movement to determine the potential for breaching of the dam or impact on the seepage control system.

(c) Consider damage to drainage layers or structural components with low strain capacity. In the case where limited deformation cannot be tolerated, a poor performance level can be assigned without considering the actual magnitude of post-failure deformation.

f. Example Problems.

(1) A detailed explanation is provided in Appendix D on how to perform a reliability index analysis of a simple slope using the infinite slope analysis method.

(2) A steady state problem is presented in Appendix I to demonstrate how to use the method on a more complex, real world problem.